01 May 2013, 2:00 pm - 4:00 pm

Geotechnical Investigation at Barrage for Foundation Design at Nyamjang Chhu Hydro Power Project, Tawang, Arunachal Pradesh, India — A Case Study

Ram Bachan Shivali
Indo Canadian Consultancy Services, India

Rakesh Mahajan
Indo Canadian Consultancy Services, India

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Seventh International Conference on Case Histories in Geotechnical Engineering
April 29 - May 4, 2013 CHICAGO
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GEOTECHNICAL INVESTIGATION AT BARRAGE FOR FOUNDATION DESIGN AT NYAMJANG CHHU HYDRO POWER PROJECT, TAWANG, ARUNACHAL PRADAESH, INDIA - A CASE STUDY

Ram Bachan Shivali
Dy. Chief Principal Engineer (Civil) & ex.SRO (CSMRS) Min.WR
Indo Canadian Consultancy Services
Bhilwara Tower, A-12, Sect.-1, NOIDA-201301, India
E-mail: rambachan.shivali@lnjbhilwara.com , M:+91-9968629778
Tel: +91-120-4390300, Extn.:326, Fax:+91-120-2531648,2531745

Rakesh Mahajan
Director (Design Civil)
Indo Canadian Consultancy Services
Bhilwara Tower, A-12, Sect.-1, NOIDA-201301, India
E-mail: rakeshmahajan@lnjbhilwara.com , M:+91-9810504919
Tel:+91-120-4390300,Extn.:434,Fax:+9-120-2531648,2531745

ABSTRACT

The Nyamjang Chhu Hydro Electric Project (780 MW) located in District Tawang, Arunachal Pradesh, India, proposes construction of a barrage across Nyamjang Chhu River for diversion of water for power generation. The diversion barrage comprises of gated spillway and sluiceway, head regulator, surface desanders and a collection pool cum tunnel intake.

Geotechnical investigations for assessment of subsoil strata at barrage location and its evaluation for foundation design included drilling of 13 no boreholes totaling 551m. In-Situ permeability and standard penetration tests (SPT) were conducted in the boreholes along with Dynamic Core Penetration Test (DCPT) conducted adjacent to boreholes. Disturbed and undisturbed soil samples were also collected for conducting laboratory tests. The laboratory testing included Mechanical Analysis, Atterberg's Limits, Specific Gravity, In-Situ Density, Tri-axial Shear test under Consolidation Un-drained Condition (CU) and Consolidation test. Rock core samples were tested for Density, Sp. Gravity, Uniaxial Compressive Strength (UCS), Modulus of Elasticity and Poison's Ratio.

The field SPT values (N_{SPT}) in 9 boreholes were observed in the range of 4 to 59 at different depth. In 2 boreholes, N_{SPT} values were observed in the range of 41-72. In remaining 2 boreholes, SPT was not carried out as the foundation stratum comprises of boulders and rock. The soil stratum in Barrage area indicates presence of medium to fine & very fine sand of varying compactness. The 'N_{SPT}' values observed in the boreholes were corrected for overburden pressure and dilantancy. Permeability tests indicate permeability value in the range of 10^{-2} to 10^{-3} cm/sec in upper 6 to 9 m depth in majority of the boreholes indicating presence of pervious sandy soil. A few permeability values were observed in the semi pervious range of 10^{-4} to 10^{-5} cm/sec.

Dynamic Cone Penetration Test (DCPT) at 10 locations was conducted adjacent to boreholes to verify the N_{SPT} test results. DCPT 'N' values in the range of 22 to 59 were observed with refusal depth varying from 5.10m to 12.30m. The DCPT 'N' values were converted to their corresponding N_{SPT} values and the corrected SPT 'N' values and converted DCPT 'N' values are also compared. The project is located in earthquake prone area with MCE and DBE values of 0.288g and 0.197g for an earthquake magnitude of 7.7 (Mercalli Scale).

In view of the significantly low SPT N values observed during field testing, analysis for liquefaction potential at barrage location was undertaken based on simplified approach suggested by Seed & Idriss (1982 & 1999) to evaluate liquefaction potential of sandy foundation strata. The approach involves computation of Cyclic Stress Ratio (CSR) τ_{av}/σ'_v and comparing it with the Cyclic Resistance Ratio (CRR) given by Seed & Idriss corresponding to earthquake of same magnitude. The CSR value at various depths in each bore hole is also plotted against the critical boundary limit for liquefaction provided by Seed & Idriss. Factor of safety against liquefaction is then calculated as ratio of CRR to CSR with value less than and equal to 1 indicating liquefaction potential of sandy soil.

Based on observed penetration resistance, foundation areas having 'N_{SPT}' value lower than 40 are assessed to be liable for liquefaction and such areas are proposed to be treated by Vibro-compaction method to make the sand dense thereby eliminating the possibility of liquefaction due to development of large strains under high pore water pressure.

INTRODUCTION

The Nyamjang Chhu Hydro Electric Project proposes to construct a Barrage across river Nyamjang Chhu having 742 meters length at top and having a maximum height of 15 meters above river bed. The Barrage is founded on strata comprising mainly of medium to fine sand and silt.

The field and laboratory investigation at Barrage and Intake areas included borehole drilling for assessment of strata variation and its evaluation. Permeability and SPT were carried out in the boreholes and DCPT was carried out on the surface close to the drilled boreholes in the barrage area. Undisturbed sampling in borehole was not possible due to presence of sand mixed gravels and cobbles /boulders at various depths.

A total 13 Nos. of boreholes in year 2011 were drilled for geotechnical assessment of foundation strata. Field investigation included permeability, Standard Penetration Test (SPT) and Dynamic Cone Penetration Test (DCPT). The depth of boreholes for SPT varied upto 40.5m and for DCPT refusal depth varied from 5.10 to 12.30m. Bed rock was encountered in 3 boreholes and only percolation test was carried out in the rocky strata.

Layout Plan of Barrage showing borehole location is shown below as exhibit-1. Details of field investigation are discussed below.

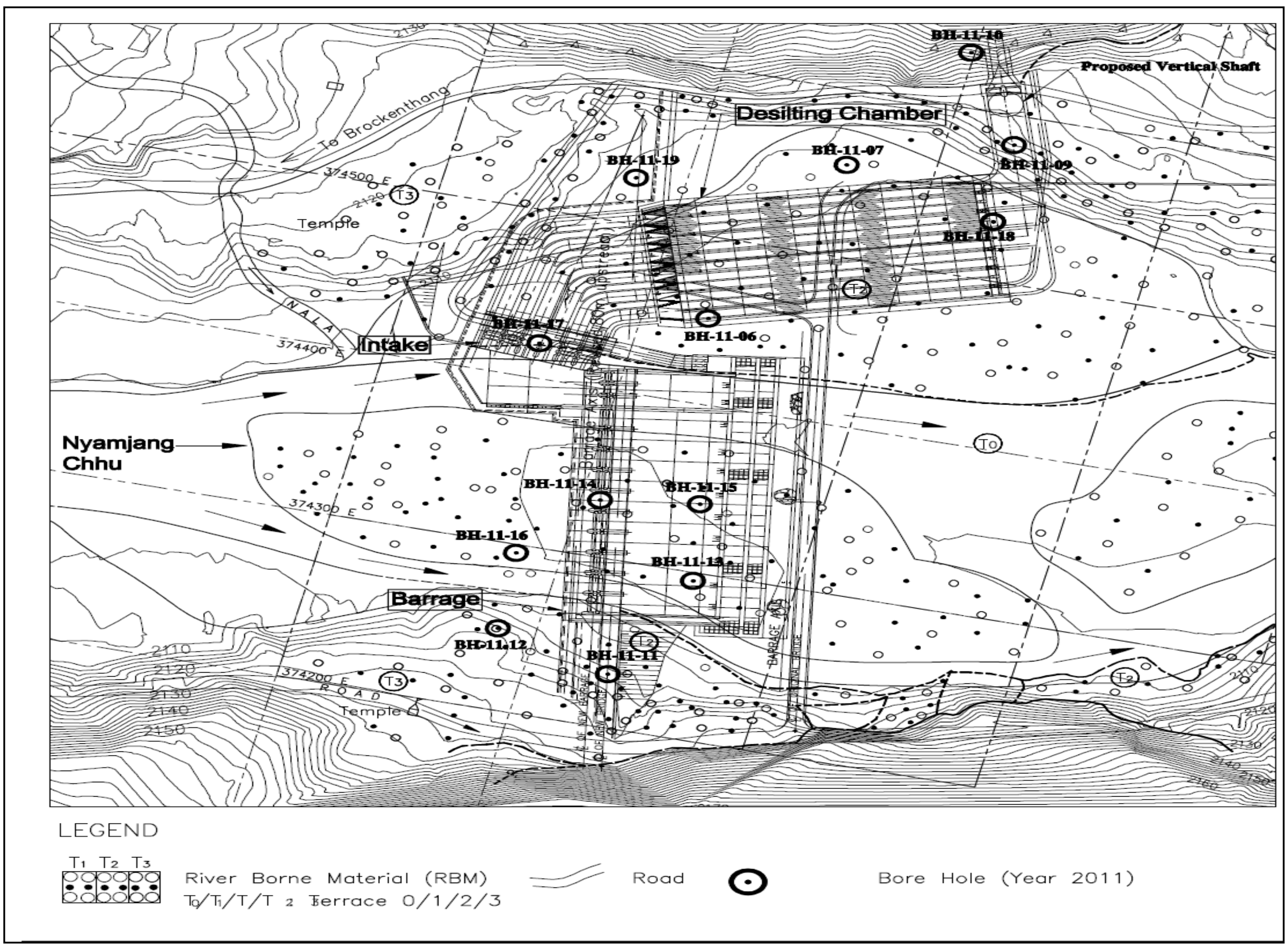

Exhibit-1: Layout Plan of Barrage showing borehole location

Actual site photograph of proposed barrage location is shown in exhibit-2

Exhibit-2: Site photograph at Barrage and Intake location

GEOTECHNICAL INVESTIGATION

Drilling

The boreholes were drilled at identified locations on both banks of river as well as in the flow channel. Initial drilling was done with Hx size in loose strata with a steel pipe casing. Subsequent drilling was done with Nx size by using single, double and triple barrel as per requirement of strata and to achieve good core recovery. Each drilling run was simultaneously followed by casing down with casing bit, test for permeability and SPT.

Borehole information like Northing and Easting coordinate, Elevation and position, RDs and field observation including seepage, water table, SPT value, change of color and artesian water zone if any during the drilling of each borehole was recorded on the standard format in field book. Drilling was performed as per Bureau of Indian Standards (BIS) Codes like- IS:1892-1997, IS:6926-1996, IS:4453-1990, IS:4078-1980, IS:4464-1995 and IS:5313-1980 Guides for Core Drilling and Observations. Drilling cores were preserved in wooden core boxes. Photograph of all core boxes were also taken (exhibit-3).

Exhibit-3: Log of Soil sample from Boreholes at Barrage and Intake area

Standard Penetration Test

The SPT was conducted as per Bureau of Indian Standards (BIS) Code No.-IS: 2131-1997 Method of Standard Penetration Test for soils equivalent to the ASTM Standard code ASTM D 1586-08a and Split Spoon Sampler IS 9640-1980.

The procedure adopted included drilling with H_x and N_X size barrels using casing. The barrel is lowered up to desired test depth and the borehole & barrel is washed with the help of centrifugal pump. In the drilling assembly, a dropping hammer is used to carry out the SPT. Weight of dropping hammer is 63.5 kg and height of drop is 75 cm. Penetration readings are noted for every 15 cm depth upto total depth of 45cm. N_{SPT} values corresponding to the last 30 cm (45-15) penetration is considered as penetration resistance.

Refusal is considered when the number of blows exceeds 50 for 15 cm penetration. Similarly, if the sampler sinks under its own weight in case of very soft soil, N_{SPT} is taken as zero.

The photograph of SPT Sampler tube is shown in exhibits-4.

Exhibit-4: SPT Sampler Tube

In-Situ Permeability Test/Percolation Test

In-situ permeability tests were carried out using constant head method in both soil/overburden as well as in rock. Permeability test is carried out at every 3 m interval in the borehole. Observations are noted after every 5 minutes interval till 3 constant readings are observed at each depth.

The following basic formula is used for calculation of permeability value in soil/over burden:

$$k = \frac{Q}{5.5rH} cm/\sec \qquad (1)$$

Where,

k = Coefficient of Permeability in cm/sec.
Q = Discharge (rate of water loss) in cm3/sec.
r = Radius of borehole in cm
H = Water head in cm

In-situ permeability test in rock is conducted by the use of a single or double rubber packer. The packers are pressurized to a higher value than the water pressure developed in the test section to ensure proper sealing. Permeability is calculated from the following equation:

$$k = \frac{Q}{2\pi LH} \log_e (L/r) \ cm/\sec \qquad (2)$$

Where,

k = Coefficient of Permeability in cm/sec.
Q = Discharge (rate of water flow) in cm3/sec.
L = Test section (uncased section) in cm
r = Radius of borehole in cm
H = Differential water head in cm

The Permeability test was conducted as per Bureau of Indian Standards (BIS) Code No.-IS: 5529-1985 (Pt.-2) and ASTM 5930. The water level measurement in drill hole was carried out as per IS: 6935-1998 and ASTM D 4750.

Dynamic Cone Penetration Test (DCPT)

The DCPT test is used for finding depth to hard stratum and to have an approximate indication of the strength and other properties of cohesion less soils from which undisturbed sampling was not possible. DCPT tests were performed at 10 locations for correlation with SPT test carried out at adjacent location. It was observed that refusal depth varies from 5.10 m to 12.6 m and observed penetration resistance (N-value) varies from 22 to 59.

The DCPT test was carried out with same assembly of Tripod Drilling as in SPT by continuously hammering the cone driven by a weight of 63.5 kg which is allowed to fall free from a height of 75 cm on a driving head attached to drill rods. Penetration readings were noted at every 30cm depth (in 3 steps of 10cm each) and continued up to refusal depth when the number of blows exceeds 20 for 10 cm penetration.

The DCPT test was conducted as per Bureau of Indian Standards (BIS) Code No.:4968 (PART-3)-1987 Method of Dynamic Cone Penetration Test for soils.

The Photograph for DCPT Testing work assembly is shown in exhibit-5. DCPT N-Value observed adjacent to boreholes of SPT and the correlation of depth wise SPT N-value & DCPT N-value is presented in exhibit-6.

Exhibit-5: DCPT testing assembly

DCPT N-Value on the surface near Borehole location				SPT N-Value in Borehole corresponding at aprox. depth of DCPT and converted DCPT N-Value into SPT N-Value					
Borehole No.	Max. Depth (m)	Max. Observed N-Value	Corrected DCPT N-Value	Borehole No.	Depth (m)	Max. Observed N-Value	Corrected N-Value	Observed DCPT N-Value Converted into SPT N-Value	Corrected DCPT N-Value Converted into SPT N-Value
BH-11-06	5.1	42 at 4.2 m	27	BH-11-06	4.5-6.0	NO SPT	0	**24**	**16**
					18.0	42	28		
BH-11-07	5.1	34	22	BH-11-07	3 and upto 25.5 no test	47	41	19	13
					25.5	71	43		
BH-11-09	Not done			BH-11-09	3	32			
BH-11-10	Not done due to collovium			BH-11-10	Not done due to collovium				
BH-11-11	Not done			BH-11-11	16.5	16	14		
BH-11-12	7.5	59	38	BH-11-12	7.5	12	12	33	22
					40.5	26	16		
BH-11-13	9.6	59	38	BH-11-13	10.5	11	11	33	22
					31.5	39	20		
BH-11-14	12.3	56	36	BH-11-14	13.5	17	15	32	21
					40.5	63	24		
BH-11-15	8.7	61	40	BH-11-15	7.5	18	17	35	23
					37.5	59	24		
BH-11-16	9.3	58	38	BH-11-16	10.5	18	16	33	22
					40.5	59	23		
BH-11-17	5.1	45 at 3.3m after that	29	BH-11-17	4.5	14	14	30	20
					40.5	22	13		
BH-11-18	5.1	31	20	BH-11-18	4.5	14	14	18	11
					19.5	16	14		
BH-11-19	9.6	57	37	BH-11-19	10.5	24	19	33	21
Note: From the above test result data, it was found that on average DCPT N-Values are 3-5 times higher than SPT N-Value									

Exhibit-6: Correlation of depth wise SPT N-value & DCPT N-value

Laboratory Testing

The following laboratory investigations were carried out on the soil samples as per the Bureau of Indian Standards IS 2720 (Part III), (Part IV), (Part V), (Part VII), (Part XII), and (Part XVII): Specific Gravity, Triaxial shear test, consolidation test, bulk density, dry density and moisture content.

A total of 86 Nos. of disturbed soil sample from different depth varying up to 40.5m were collected and tested.

Specific Gravity test carried out on samples indicates an average value of 2.62.

Tri-axial Shear Tests under Consolidated Un-Drained Condition (CU) for Undisturbed (UDS) samples indicate an average value as follows:

Total Cohesion Value (c)	= 34.64 KPa
Total Friction of Angle (φ)	= 30^0
Effective Cohesion Value (c')	= 32.07 KPa
Effective Friction of Angle (φ')	= 33.7^0

Consolidation test indicates an average value as follows:

Compression Index (Cc):	= 0.16
Pre-consolidation pressure	= 90 KPa

Moisture content, Bulk Density (Moist/Saturated), and Dry Density value are observed as follows:

Moisture Content	= 14.75 %
Bulk Density	= 1.69 g/cc
Dry Density	= 1.45 g/cc

METHODOLOGY FOR ASSESSMENT OF LIQUEFACTION POTENTIAL

During last 40 to 45 years, liquefaction of sandy strata caused due to earthquake has been studied extensively by hundreds of researchers around the world. The disastrous consequences of liquefaction were brought to the fore in 1964 by the experience due to earthquake at Nigata, Japan, where large buildings slowly rolled over on their sides, and pipes and tanks floated to the surface through the temporarily fluidised soil in which they were buried. Liquefaction is also known to trigger earth slides and large displacements of earthen dams. Kobe earthquake demonstrated how port and harbour structures suffer damage due to liquefaction. In India states like-Maharastra, Jammu & Kashmir, Uttrakhand and North Eatern state of Sikkim, Arunachal Pradesh have reported costly damage due to liquefaction of sands and silty clays caused by earthquakes.

The term "liquefaction" describes the situation where pore water pressure builds up and approaches a value equal to the applied confining pressure or overburden pressure. The effective stress becomes zero and the sand begins to undergo excessive deformations or may begin to flow. The state when the pore water pressure equals the confining stress has been called initial liquefaction. This phenomenon is known as earthquake induced liquefaction of sand. Liquefaction susceptibility of the sandy soil depends on its initial formation state (stress and density characteristics at the time of earthquake) as it dictates the tendency to generate pore pressures during cyclic loading If the sand is loose, it will undergo large deformations with shear strains that may exceed 20% or may flow. If the sand is dense, it may not develop large strains even though pore water pressure rise may be equal to the confining pressure.

Geotechnical Investigation for Nyamjang Chhu project indicates loose foundation materials. As the project lies in a relatively high seismic zone, the possibility of liquefaction during earthquake loading has been evaluated.

First step in liquefaction hazard evaluation usually is to analyze liquefaction susceptibility. There are several criteria to analyze liquefaction susceptibility including:

- Historical Criteria
- Geologic Criteria
- Compositional Criteria
- State Criteria

The phenomenon of liquefaction has been described in the following simplified manner by Seed, "As a consequence of the applied cyclic stresses, the structure of the cohesionless sand tends to become more compact with a resulting transfer of stress to the pore water and a reduction in stress on the soil grains". Seed and Idriss (1982 & 1999) have proposed a simplified procedure for evaluation of liquefaction potential. The procedure involves calculating stresses induced due to earthquake ie Cyclic Stress Ratio (CSR) and comparing it with soil strength ie Cyclic Resistance Ratio (CRR) which can be obtained either from the laboratory tests or from field tests. In the present study, stresses were calculated using the simplified procedure and strength of the soil is determined using field Standard Penetration Test (SPT).

Stresses induced by Earthquake

The stresses induced by earthquake in the soil are calculated by assuming the soil column above a soil element at depth 'h' as a rigid body (exhibit 7). In reality the soil column is not a rigid body and behaves in a very non-linear fashion. Therefore, Seed and Idriss introduced stress reduction factor (r_d) to account for the fact that the soil column is a deformable body. This relationship was graphically presented between depth and r_d value. If the maximum ground acceleration is α_{max}, the maximum shear stress on the soil element would be

$$(\tau_{max}) = \gamma.h/g.(\alpha_{max}) \qquad (3)$$

Where, γ is the unit wt. of the soil.

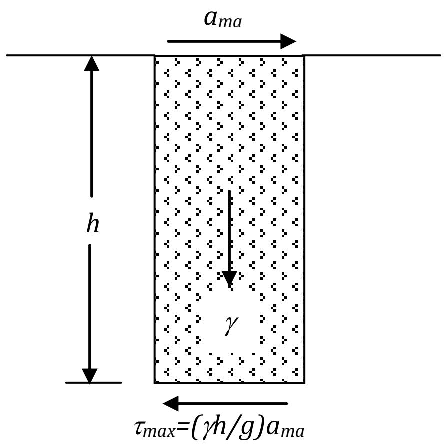

Exhibit-7: Maximum Shear Stress at a depth for a Rigid Soil Column

Because the soil column behaves as a deformable body, the actual shear stress $(\tau_{max})_\alpha$ at a depth 'h' will be less than (τ_{max}) and given by

$$(\tau_{max})_\alpha = r_d . (\tau_{max})$$

$$(\tau_{max})_\alpha = r_d . \gamma.h/g. \alpha_{max}$$

$$r_d = (\tau_{max})_\alpha/(\tau_{max}) \quad (4)$$

Where, r_d is a stress reduction coefficient. The value of r_d for wide variety of earthquake motion and sandy soil condition is given in exhibit-8 (Seed and Idriss).

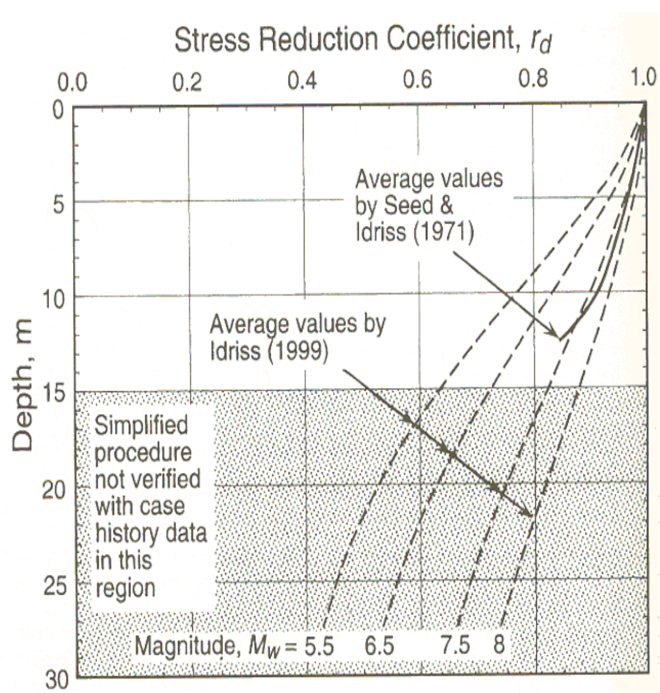

Exhibit-8: Stress Reduction Factor at different depth corresponding to varying earthquake magnitude

In Nyamjung Chhu Project, the stress induced by earthquake having acceleration of 0.288g and 0.192g for the Maximum Credible Earthquake (MCE) and Design Basis Earthquake (DBE) respectively (as recommended by department of Earthquake Engineering, IIT, Roorkee) has been used for developing a project specific curve for soil depth versus stress reduction factor (rd) corresponding to an earthquake of magnitude of 7.7 (exhibit-9).

The actual time history of shear stresses at any point in a soil deposit during an earthquake will have an irregular form. From such relationships, it is necessary to determine the equivalent average shear stress. Based on research in a number of different cases, it has been found that with a reasonable degree of accuracy, the average equivalent uniform shear stress τ_{av} is about 65% of the actual shear stress $(\tau_{max})_a$.

$$\tau_{av} = 0.65 (\tau_{max})_a$$

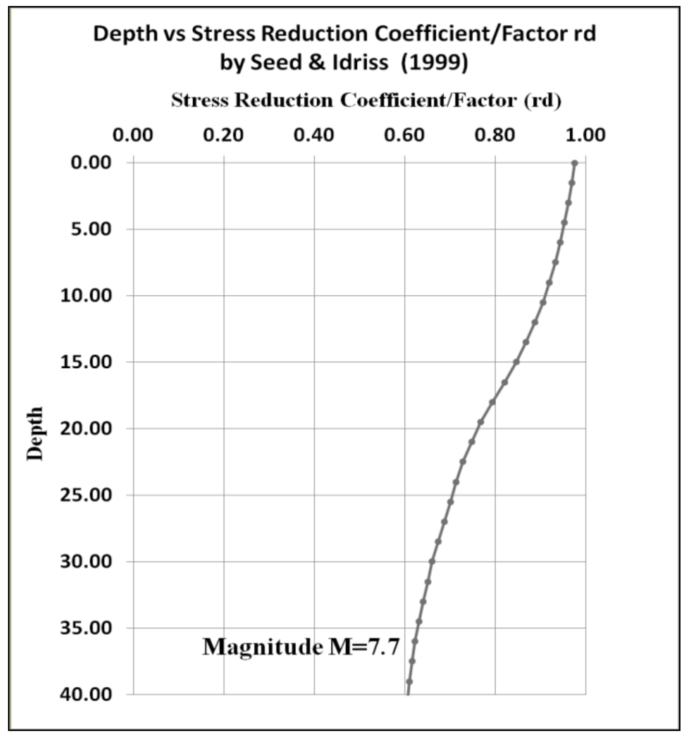

Exhibit-9: Range of Stress Reduction Coeff. (r_d) Vs. Depth at Earthquake magnitude =7.7

$$\tau_{av} = 0.65 \frac{\gamma h}{g} a_{\max} r_d \quad (5)$$

CSR is defined as the ratio of the average cyclic shear stress induced by earthquake to the initial vertical effective overburden stress. The average cyclic shear stress is determined as follows:

$$\text{CSR} = \tau_{av}/\sigma'_v = 0.65\ \gamma.h/g.\ \alpha_{max}.\ r_{d/}\ \sigma'_v \quad (6)$$

The Cyclic Stress Ratio (CSR) (τ_{av}/σ'_v) based on average shear stress and effective overburden stress induced at various depths were calculated for each borehole for Nyamjang Chhu Project.

Soil Resistance during Earthquake Loading

After characterizing the earthquake loading, the next important step for evaluation of liquefaction susceptibility, is to estimate the resistance offered by the soil to cyclic shear stresses induced during earthquake loading. In 60's and 70's articles from various researchers dealing exclusively with laboratory investigations to study various factors influencing the liquefaction susceptibility of soils (void ratio, amplitude of peak pulsating stress, frequency of load application etc.) were published. For this project, estimation of soil strength based on field tests data provided by SPT and DCPT is used.

Seed and Idriss have developed a chart indicating the limiting value of Cyclic Stress Ratio at various SPT N-values (exhibit-10) at which liquefaction occurs. This limiting CSR value is also defined as Cyclic Resistance Ratio (CRR). The chart developed by Seed and Idriss is applicable only for earthquake magnitude 7.5. Seed & Idriss and other researchers have suggested magnitude scaling factor (MSF) (exhibit 11 & 12) for applying to CRR value to determine the corrected CRR value at different magnitude of earthquake. Corresponding to earthquake of magnitude 7.7 for Nyamjang Chhu Project the applicable factor is 0.975. The modified chart of limiting CSR Vs.SPT N-value for earthquake of magnitude 7.7 for Nyamjang Chhu project is also indicated in exhibit-10.

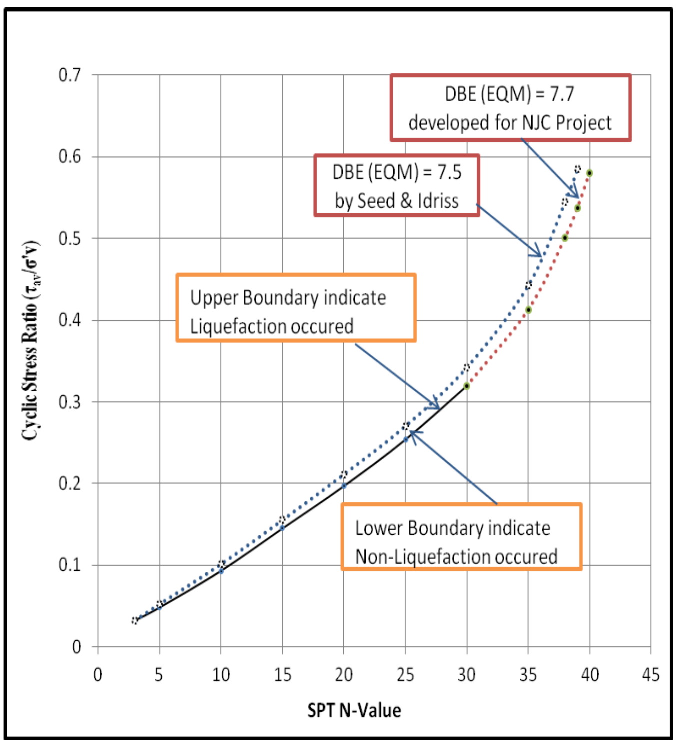

Exhibit-10: Critical limit boundary for liquefiable and non-liquefiable sand at earthquake magnitude 7.5 & 7.7

Exhibit-11: Magnitude Scaling Factor Values as defined by Various researchers

Magnitude (M)	Seed and Idris	Idriss	Ambraseys	Arango		Andrus and Stokoe	Youd and Noble		
				Distance based	Energy based		$P_L<$ 20%	$P_L<$ 32%	$P_L<$ 50%
5.5	1.43	2.20	2.86	3.00	2.20	2.8	2.86	3.42	4.44
6.0	1.32	1.76	2.20	2.00	1.65	2.1	1.93	2.35	2.92
6.5	1.19	1.44	1.69	1.60	1.40	1.6	1.34	1.66	1.99
7.0	1.08	1.19	1.30	1.25	1.10	1.25	1.00	1.20	1.39
7.5	1.00	1.00	1.00	1.00	1.00	1.00	-	-	1.00
8.0	0.94	0.84	0.67	0.75	0.85	0.8?	-	-	0.73?
8.5	0.89	0.72	0.44	-	-	0.65?	-	-	0.56?

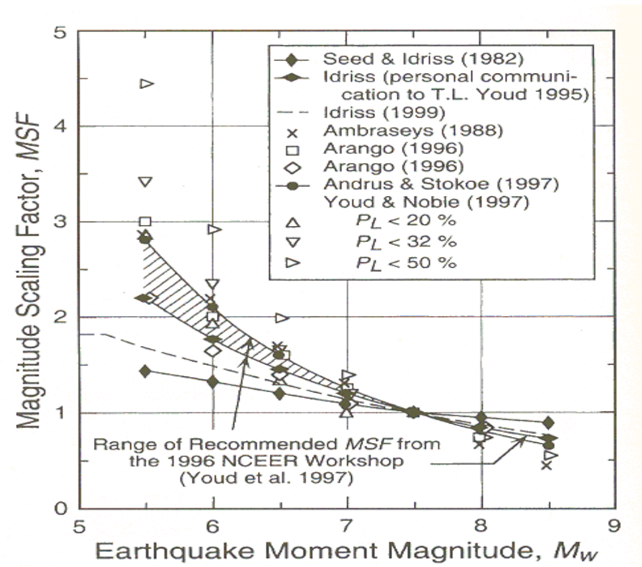

Exhibit-12: MSF and Earthquake Moment Magnitude (Mw)

Assessment of Factor of Safety against Liquefaction

The ratio of CRR and CSR determines the Factor of Safety against Liquefaction. Liquefaction Potential exists or the foundation strata is Liquefiable (L) if FOS is equal to or less than 1.0 and Non- Liquefiable (NL) if FOS is more than 1.0. Based on the assessment of liquefaction potential, the depth for foundation treatment is determined to keep the barrage safe against earthquake loading.

CASE STUDY

Liquefaction analysis was performed for Nyamjang Chhu project based on test results of total 11 nos of boreholes at the proposed barrage structure. Result of this analysis indicate that soil condition in 6 nos. of boreholes is in liquefiable (L) zone, 2 nos. of boreholes are in Non-Liquefiable zone and 3 nos. of boreholes lie in semi Liquefiable zone. Typical computational details for Liquefaction potential in one of the borehole BH-11-15 is given in Exhibit-13. The Graphical representation of

Liquefiable and Non-Liquefiable foundation strata of 11 boreholes is shown in Exhibits-14-24

Exhibit-13:Computational Sheet of Liquefaction Potential of BH-11-15

NJC Hydro Power Project Limitd, Tawang Arunachal Pradesh						
				Water Table =0.60m		
Bore Hole-BH-11-15				α_h=0.288g or α_{max} =0.288g		
				α_v=0.192g DBE=7.7		
Depth (m)	Corected SPT N value due to over burden,fine sand and silt & Deliantancy	Shear Stess Induced τ_{av} (kg/cm^2)	Cyclic Stress Ratio (CSR) caused by Earth Quake	Cyclic Resistance Ratio (CRR) a critical bondary for L & NL from N-Value for soil strength from Liqu.graph/chart	FOS against liquefaction	Liquefaction Potential (L-Liquefiable , NL-Non Liquefiable)
1.50						
4.50						
7.50	18	0.221	0.383	0.176	0.460	L
10.50	20	0.301	0.383	0.197	0.514	L
13.50	20	0.370	0.373	0.197	0.527	L
16.50	26	0.428	0.357	0.267	0.748	L
19.50	21	0.473	0.337	0.208	0.618	L
22.50	22	0.518	0.321	0.219	0.681	L
25.50	17	0.565	0.310	0.166	0.535	L
28.50	26	0.607	0.299	0.267	0.892	L
31.50	25	0.648	0.290	0.254	0.876	L
34.50	24	0.688	0.282	0.241	0.855	L
37.50	28	0.730	0.276	0.293	1.063	NL
40.50	14	0.775	0.272	0.135	0.497	L

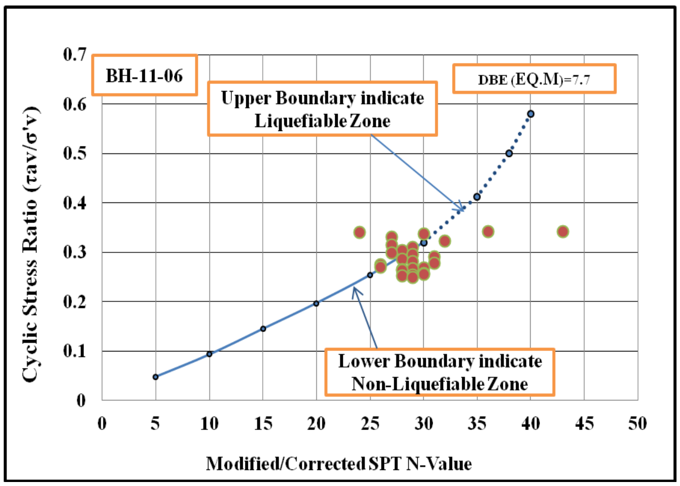

Exhibit-14: Liquefaction Potential of BH-11-06

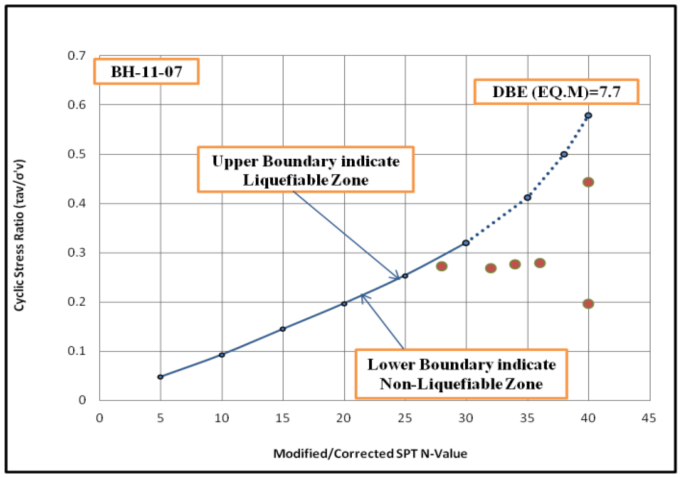

Exhibit-15: Liquefaction Potential of BH-11-07

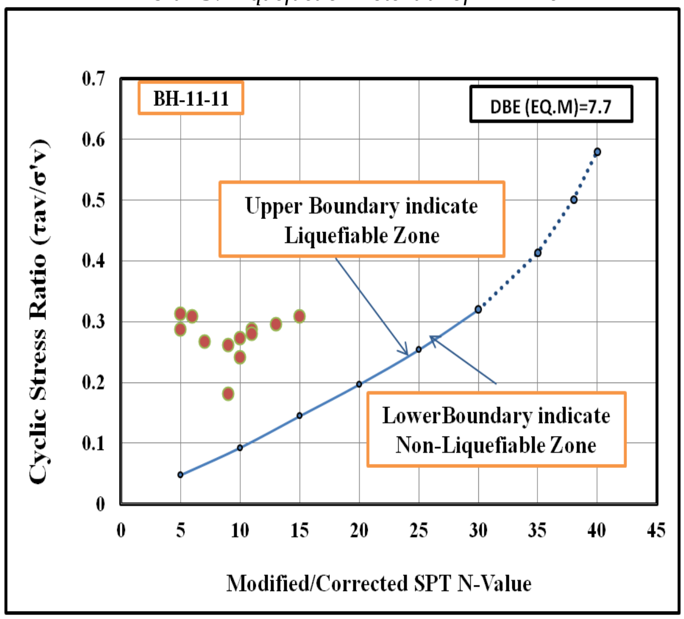

Exhibit-16: Liquefaction Potential of BH-11-11

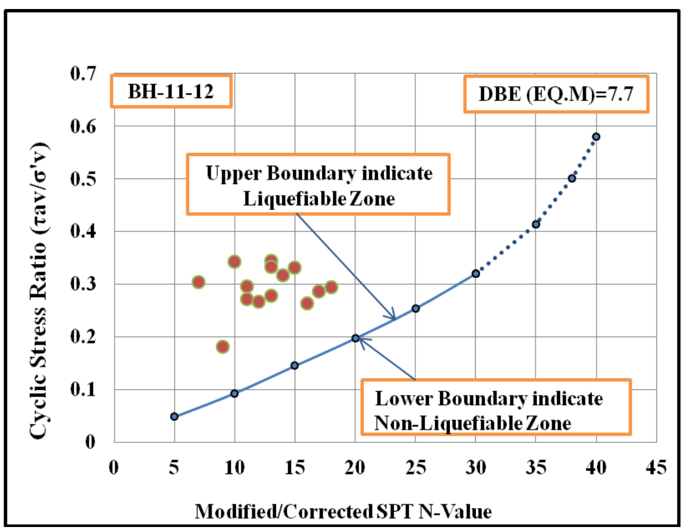

Exhibit-17: Liquefaction Potential of BH-11-12

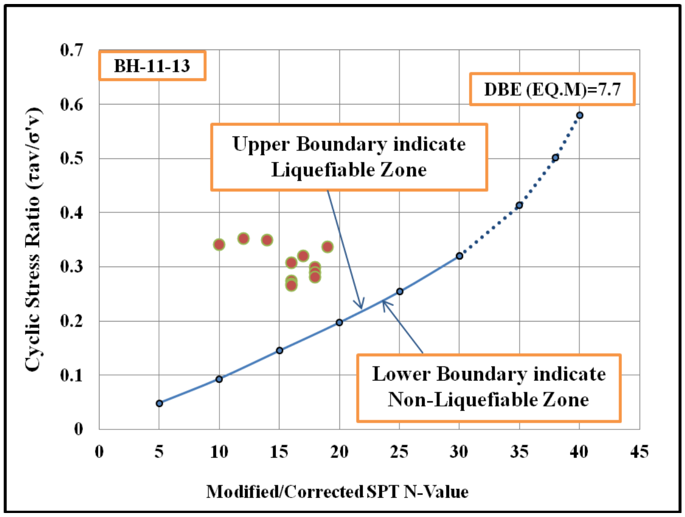

Exhibit-18: Liquefaction Potential of BH-11-13

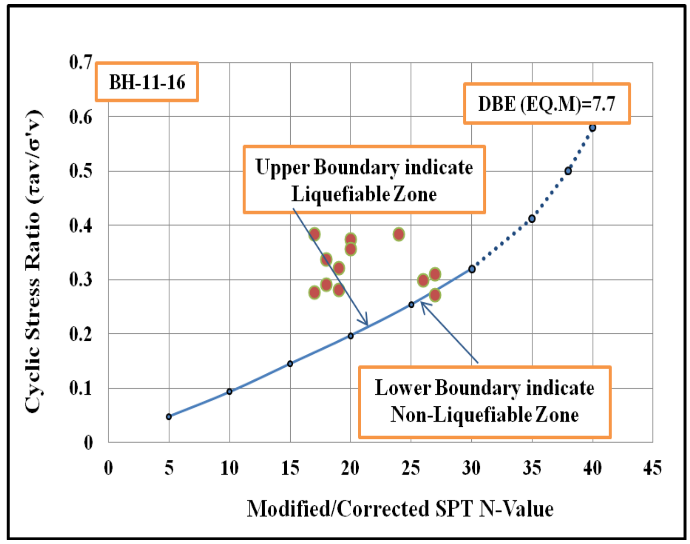

Exhibit-21: Liquefaction Potential of BH-11-16

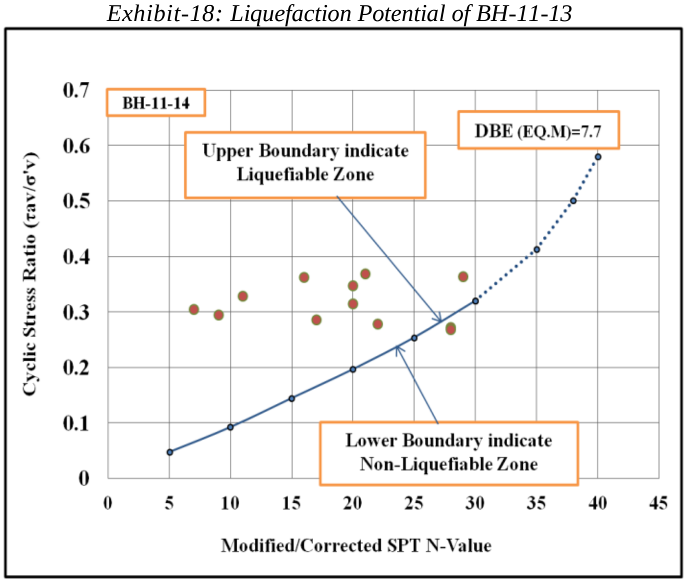

Exhibit-19: Liquefaction Potential of BH-11-14

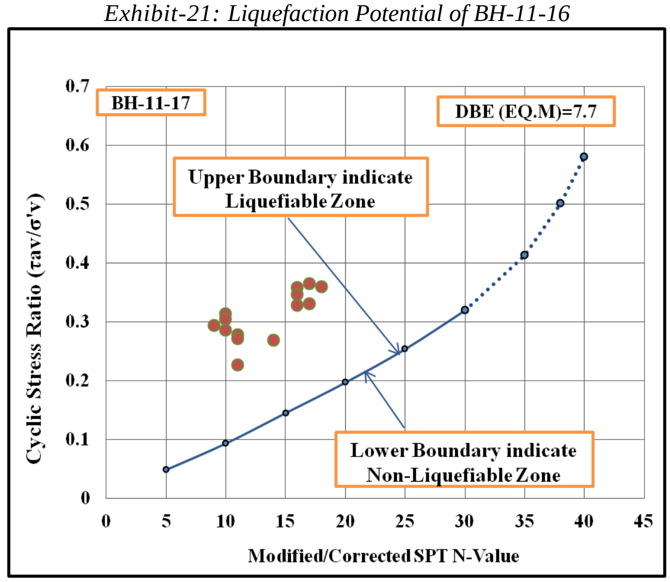

Exhibit-22: Liquefaction Potential of BH-11-17

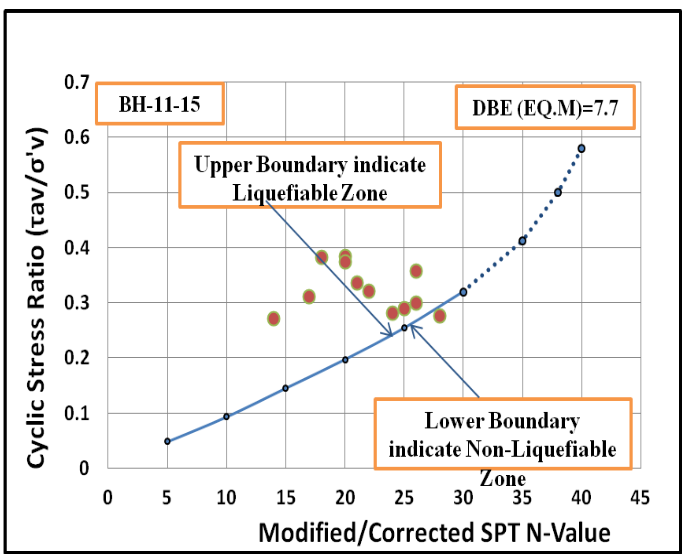

Exhibit-20: Liquefaction Potential of BH-11-15

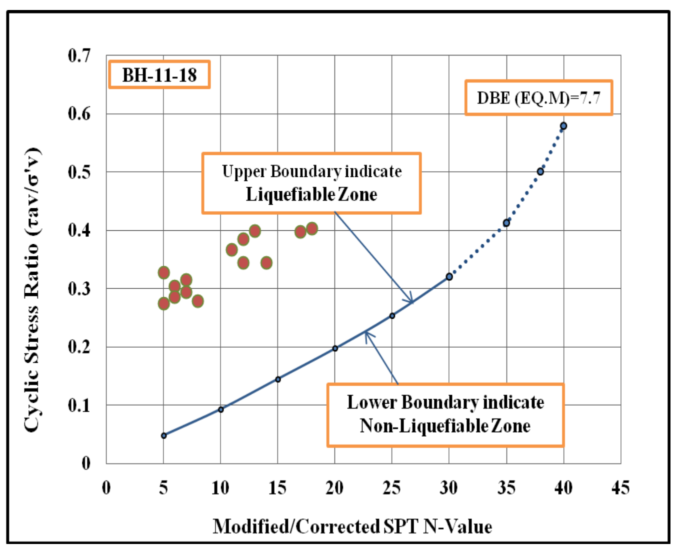

Exhibit-23: Liquefaction Potential of BH-11-18

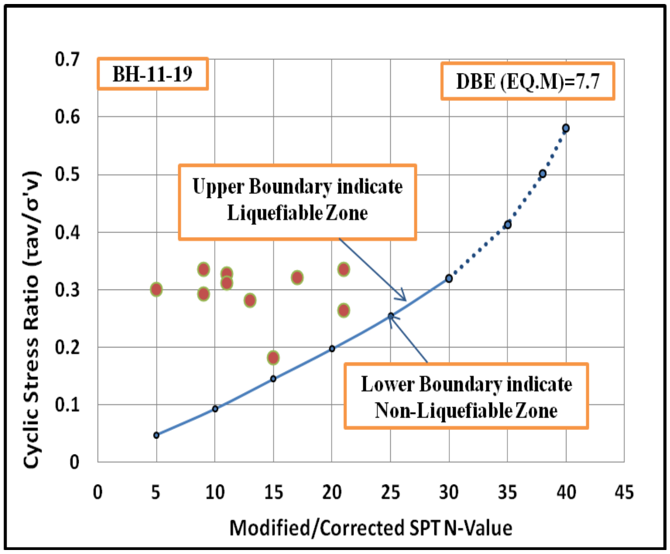

Exhibit-24: Liquefaction Potential of BH-11-19

The procedure for assessment of liquefaction potential can be summarized in following steps:

i) Establish the design basis earthquake, and obtain peak ground acceleration. Also obtain number of significant cycles corresponding to earthquake magnitude.
ii) From field SPT N-value determine the limiting CSR ratio or Cyclic shear Resistance Ratio (CRR) from corrected graph of CSR Vs. N-Value after applying magnitude scaling factor for determining soil strength at depth h below the ground surface.
iii) Determine the Cyclic Stress Ratio (CSR) caused by Earth Quake at depth h below the ground.
iv) At depth *h*, liquefaction will occur if ratio of CRR and CSR is less than or equal to 1.0
v) Repeat the steps (ii), (iii) and (iv) for other values of *h* to determine the liquefaction potential.
vi) Repeat steps (ii) to (v) for all boreholes.

The average Uniform Shear Stress induced as per recommendation of Seed and Idriss and Average Shear Resistance of soil with overburden pressure and dilantancy required to cause liquefaction at Nyamjang Chhu Project are shown in Exhibit-25 and limit of Liquefiable depth is also given Borehole wise in Exhibit-26.

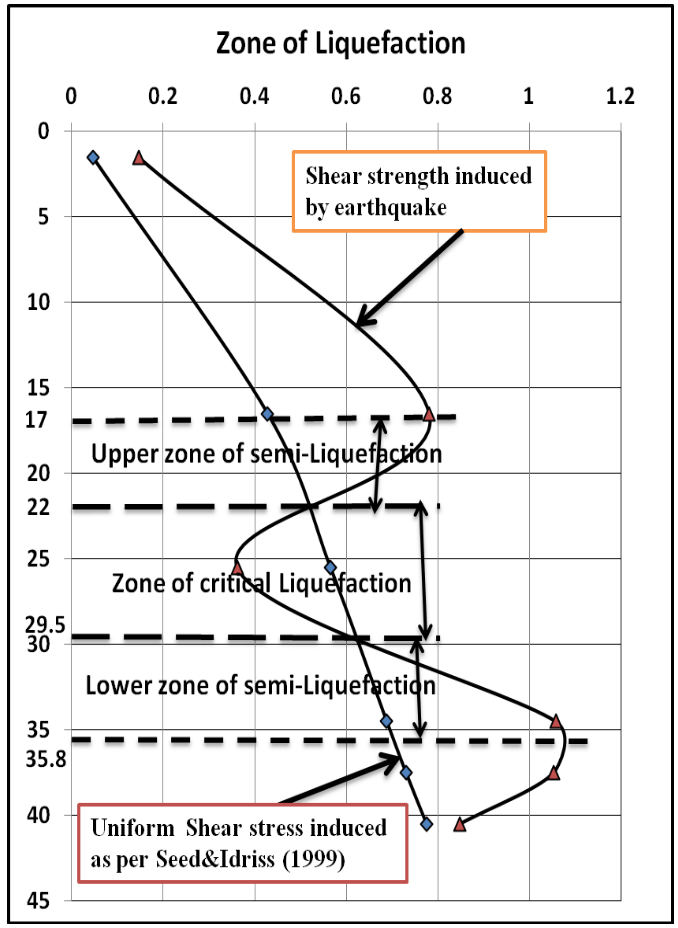

Exhibit 25: Zone of Liquefaction

BH No.	Liquefiable Depth
6	16.50
7	1.50
11	34.50
12	40.50
13	40.50
14	34.50
15	34.50
16	37.50
17	40.50
18	40.50
19	25.50

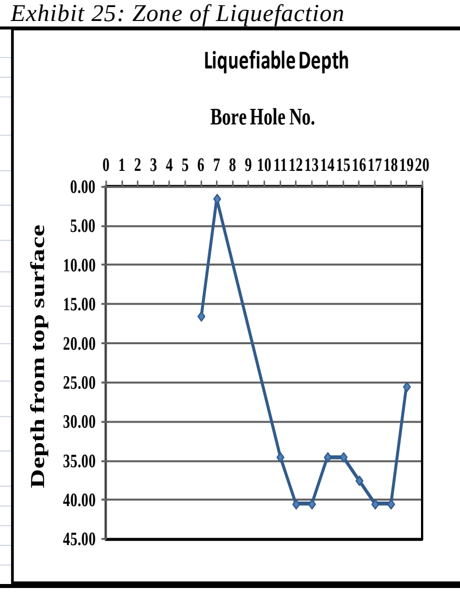

Exhibit-26: Zone of Liquefiable Depth

The literatures say that if '*N'* is above 35, the sand is in either a dense or very dense state as shown in Exhibit-27. For this condition, initial liquefaction does not produce large deformations because of the dilation tendency of the sand upon reversal of the cyclic shear stress.

Exhibit-27: Correlation between SPT *N-Value* and density of sand

SPT N Value	Description of Foundation Strata	Relative Density *(%)*
0-2	Very loose condition	0-15
2-5	Loose condition	15-35
5-20	Medium condition	35-65
20-35	Dense condition	65-85
> 35	Very Dense condition	85-100

CONCLUSION

The field Standard Penetration Test (SPT) was carried out in a total 13 nos. boreholes and SPT value in 9 Nos. boreholes was observed in the range of 4 to 59 at different depth varying from 1.5 m to 40.50 m. In 2 nos. of boreholes, SPT values were observed in the range of 41-72. The remaining two boreholes, were located in boulders and rocky strata and no SPT test was done. The soil stratum in Barrage area indicates presence of medium to fine & very fine sand of varying compactness. The SPT values observed in the boreholes were corrected for overburden pressure and dilatancy while also taking care of water table.

Dynamic Cone Penetration Test (DCPT) was conducted from the surface adjacent to 10 Boreholes to verify the actual SPT N-Value test results carried out inside the boreholes. DCPT 'N' values in the range of 22 to 59 were observed with refusal depth varying from 5.10m to 12.30m.

The observed DCPT N-values were compared SPT N value and it was observed that the DCPT N-values are 3 to 5 times greater than the observed SPT N-Values inside the boreholes.

The advantages of using the *SPT* to evaluate the liquefaction potential are as follows:

- Groundwater table: A boring must be excavated in order to perform the standard penetration test. The location of the groundwater table can be measured in the borehole, which can then be used to monitor the ground water level over time.

- Soil Type: In clean sand, the *SPT* sampler may not be able to retain a soil sample. But for most other types of soil, the *SPT* sampler will be able to retrieve a soil sample. The soil sample retrieved in the *SPT* sampler can be used to visually classify the soil and to estimate the percent fines in the soil.

The Nyamjang Chhu Project is located in earthquake prone area with MCE and DBE values of 0.288g and 0.192g for an earthquake magnitude of 7.7. The foundation condition at barrage site is analyzed based on SPT test results and assessment for liquefaction potential of foundation strata and classified as Liquefiable and Non-liquefiable. It is seen that substantial area below the barrage structure is lying in liquefiable zone. Based on observed penetration resistance trend, foundation areas having Standard Penetration Test value below 35-40 and up to critical depth varying between 25.0- 36.0m are proposed to be treated by Vibro compaction method to make the sand dense thereby eliminating the possibility of liquefaction due to development of large strains under high pore water pressure.

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